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DESIGN AND CONSTRUCTION OF THE FRENCH KING BRIDGE ON THE MOHAWK TRAIL ROUTE ACROSS THE CONNECTICUT RIVER, MASSACHUSETTS

ALBERT E. KLEINERT, JR.*

(Presented at a meeting of the Boston Society of Civil Engineers on April 18, 1933)

IN order to reduce highway accidents which have increased alarmingly in the past few years, the Massachusetts Department of Public Works has been giving increased consideration to the correction of dangerous road conditions. Narrow roads have been widened, steep grades have been reduced, dangerous curves have been corrected, grade crossings have been eliminated, dangerous intersections have been improved, narrow bridges have been widened or rebuilt, and traffic control lights and warning signals have been installed to warn motorists.

One of the most dangerous sections of road in the State, containing examples of all of the above highway problems, was the Mohawk Trail in the section through the villages of Millers Falls and Turners Falls. The old highway, beginning a little east of Millers Falls and proceeding west, is on a steep grade with a right angle turn part way down the hill where there is a blind grade crossing of the Central Vermont Railroad. A few hundred feet beyond is a second right angle turn with a narrow bridge only 15 feet wide, over Millers River, leading into the village of Millers Falls, with a third right angle turn and a 9 per cent grade leading out of the village. Beyond this point, the alignment and grade of the road is reasonably good until we reach the village of Turners Falls, where the road descends on a 7 per cent grade

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with a sharp turn at the foot of the grade, and a few hundred feet beyond a right angle turn with a sharp descent, another right angle turn, then over a canal on a narrow bridge with a railroad crossing just beyond, thence over the Connecticut River on the old suspension bridge with a roadway only 18 feet wide, at the end of which is a very dangerous right angle turn and a long steep grade uphill into Greenfield. The total length of the road is about seven miles.

Consideration was first given to changing the old road by straightening the alignment, and widening and rebuilding the bridges. One plan was to pass under the Central Vermont Railroad between the river and the present grade crossing, with a new bridge over Millers River, which would by-pass Millers Falls and would eliminate the bad

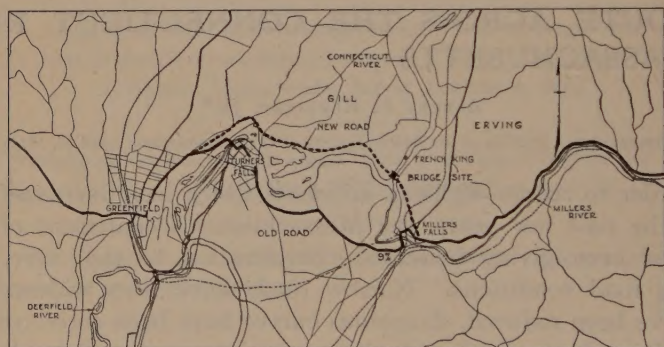


FIG. 1. — LOCATION OF FRENCH KING BRIDGE AND HIGHWAY

curvature and steep grade in the village. In Turners Falls a high bridge over the canal, the railroad and the Connecticut River was studied. This bridge, however, would have been very long and expensive, the result was not ideal, and the plan was abandoned.

Another study was developed for leaving the old highway at about the point where the new by-pass begins, but swinging to the left with a long high bridge over Millers River and the railroad yard, descending to grade in the village of Millers Falls. This would have carried the heavy travel directly through the narrow streets of Millers Falls, which was not desirable.

Consideration was also given to by-passing Turners Falls to the south with a bridge over the Connecticut River near the freight yards of the Boston & Maine Railroad, with a connection to the southerly end of the Cheapside bridge. The westerly end of this route from the Con-

necticut River to the Cheapside bridge involved railroad grade separations with steep grades and sharp curvatures, and this was accordingly abandoned.

During all this time more or less attention was being focused upon a route north of Millers Falls and Turners Falls, but the crossing of the Connecticut River, which is in a gorge with banks 150 feet above the water, presented difficulties. At first a low bridge was considered with a road winding up the side of the hills, but eventually it was decided to cross the river with a bridge at about the height of the hills, the elevation being finally fixed at 135 feet above the water.



FIG. 2.—FRENCH KING BRIDGE

On the route as finally chosen the sharpest curve has a radius of 500 feet, and the maximum grade is 6 per cent. The new road crosses the tracks of the Central Vermont Railroad by means of an overpass. There is also a grade separation where it crosses the Northfield road, which is the only important highway intersecting it. A third bridge crosses the Falls River just north of Turners Falls, and finally there is the bridge over the Connecticut River, to which this paper will be limited.

DERIVATION OF NAME

This bridge has been named the "French King bridge." For the derivation of the name one must go back to colonial days, to the time of the French and Indian Wars. Legend states that a scouting party of Indians under the command of a French officer came down the river

to locate the most advantageous point to attack the Deerfield settlement. They arrived at the rapids as night was falling and camped in the vicinity of a large rock which is near the bridge. Realizing that a knowledge of the location might be of value, the officer made a sketch of the rock and the surrounding shores, and later in the presence of his Indian followers the rock was christened in the name of His Majesty, the King of France. This version has come down from the old river men who, in driving their logs through the gorge, have not always blessed the French King boulder, as it has been the cause of numerous log jams in the river. However, the legend is to be preserved in the name of the new bridge.

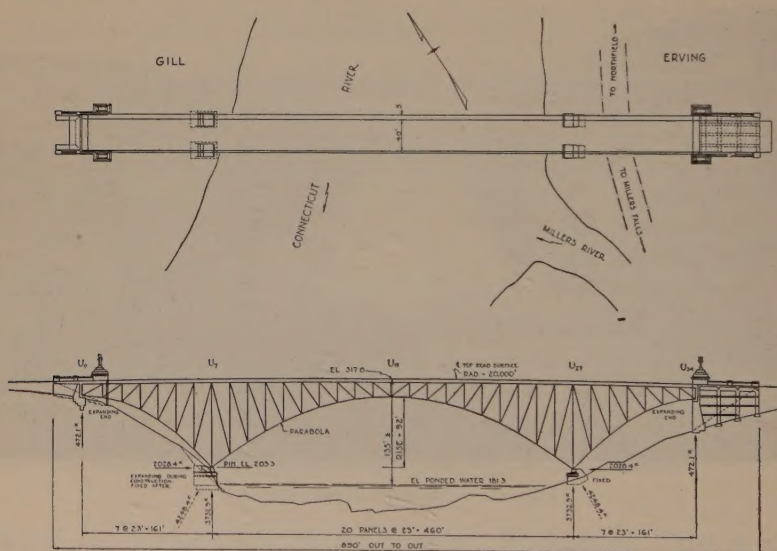


FIG. 3. — PLAN AND PROFILE

TYPE OF BRIDGE

In selecting the type of bridge to be erected upon this site, the following conditions were the governing factors: The new road was to be carried across the river at an elevation about 135 feet above the water level. The banks on each side of the river were high and steep and a narrow road had been built into the east bank at a grade approximately 30 feet above the water level. There was rock, suitable for foundations, within a few feet of the surface of the ground. The bottom of the river was uneven and rocky, and the depth of the water varied, being 34 feet at the deepest point. At this location the river

was comparatively narrow, being only 425 feet wide, and as the water was ponded for power purposes, any reduction in the waterway was prohibited. Damage to substructures by floating ice also ruled against piers in the river. With these conditions in mind, the endeavor was made to develop a bridge which not only fitted the site, but which could be erected by cantilever construction, since the deep water, rocky river bed, and the swift current discouraged the use of falsework in the river. The result of our studies is a steel deck structure, continuous over four supports, two of which are abutments placed at the ends of the bridge, high on the banks, and the other two are piers placed at the edges of the river. The superstructure is not only continuous over the four supports, but it is also fixed against horizontal movement at both of the piers. It is fixed for all loads, including the dead load of the steel itself, as definite reactions were jacked into the trusses at the conclusion of the erection. At the abutments, provision was made for the longitudinal expansion and contraction by supporting the ends of the bridge on roller nests. In naming the type of this structure we have used the term "steel continuous spandrel braced arch," because it is a steel spandrel braced arch between the piers, and is continuous to each abutment where it receives vertical support.

DESIGN OF TRUSSES

With this arrangement of the supports, the structure is indeterminate to the third degree, and therefore it was necessary to make use of some method for solving indeterminate structures in arriving at the stresses in the members of the trusses. The Method of Least Work was selected. Formulas for three of the six unknown reactions were developed in terms of the products of summations of products of the lengths, cross sectional areas, and unit load stresses, of the members of the trusses, all of which is in accordance with the theory as given in any of the complete texts on the theory of structures, such as Professor Spofford's "Theory of Structures." This method is based upon Castigliano's "Theorem of Least Work," which states that the internal work done in a stationary structure by the application of external forces is the least consistent with equilibrium. Following this principle, the equation for the internal work performed in the truss was written in terms of as many unknowns as the structure was indeterminate, differentiated with respect to each unknown, placed the equations equal to zero, and solved for the unknowns by use of determinants. The unknowns in this equation were the vertical reactions at the abutments and the horizontal reaction at the Erving pier. These were designated

as R_a , R_d , and R_c , respectively, and their values are given in the following equations:

$$R_a = \frac{lo(\sum SoSd_A^L) + mn(\sum SoSc_A^L) + p^2(\sum SoSa_A^L) - lm(\sum SoSa_A^L) - op(\sum SoSc_A^L) - np(\sum SoSd_A^L)}{klm + 2nop - kp^2 - lo^2 - mn^2}$$

$$R_d = \frac{n^2(\sum SoSd_A^L) + kp(\sum SoSc_A^L) + lo(\sum SoSa_A^L) - np(\sum SoSa_A^L) - no(\sum SoSc_A^L) - kl(\sum SoSd_A^L)}{klm + 2nop - kp^2 - lo^2 - mn^2}$$

$$R_c = \frac{kp(\sum SoSd_A^L) + o^2(\sum SoSc_A^L) + mn(\sum SoSa_A^L) - op(\sum SoSa_A^L) - km(\sum SoSc_A^L) - no(\sum SoSd_A^L)}{klm + 2nop - kp^2 - lo^2 - mn^2}$$

where —

$$k = \sum S_a^2 A_A^L$$

$$l = \sum S_c^2 A_A^L$$

$$m = \sum S_d^2 A_A^L$$

$$n = \sum S_a S_c A_A^L$$

$$o = \sum S_a S_d A_A^L$$

$$p = \sum S_c S_d A_A^L$$

S_o = the stress in any member due to external loads applied to the truss acting as a simple span, *i.e.*, with zero reactions at points *a*, and *d*, and zero horizontal reaction at point *c*.

S_a = the stress in any member due to a load of unity acting at point *a* and in the direction of R_a .

S_c = the stress in any member due to a load of unity acting at point *c* and in the direction of R_c .

S_d = the stress in any member due to a load of unity acting at point *d* and in the direction of R_d .

L = the length of the member in inches.

A = the area of the cross section of the member in square inches.

The terms k , l , m , n , o , and p may be called truss constants, for they are functions of the dimensions of the truss, and do not vary unless there is a change in the length or area of one or more of the members. Therefore these terms were evaluated for the truss, and were altered only whenever a change was made in an area.

The other summations were of products of truss constants and a stress due to external loads. Since the external loads varied in magnitude and position, these terms changed in value accordingly, and as there were several types of loads, and many positions especially for the live loads, it was decided to make influence tables for the unknown reactions, and then, with these, make influence tables for all of the members. The stress in any member for any load or system of loads was then a simple problem.

The cross-sectional areas of the members were first estimated by an approximate design for a cantilever bridge altered to provide for esti-

mated changes caused by making the trusses continuous. These areas were used in the Least Work Formulas, and a few typical members were designed to determine the approximation. Two sets of corrections were necessary before satisfactory areas were obtained. However, with an arrangement of the computations in columns, these corrections were simple, and were quickly made with but little opportunity for mistake. This method of "cut and try" proved to be much less formidable than it seemed to be at the outset of the work.

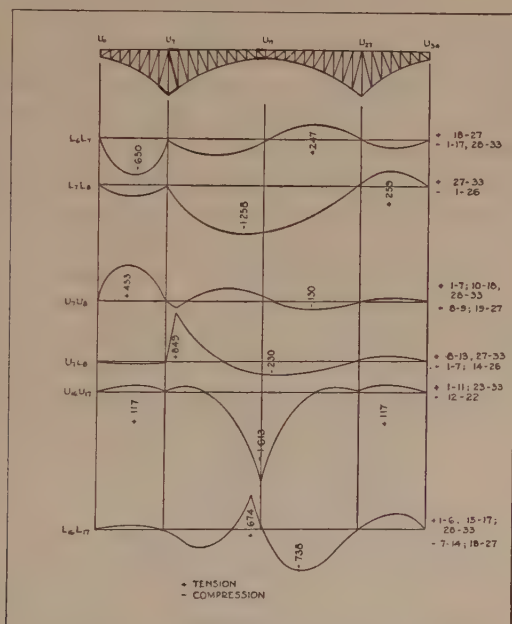


FIG. 4. — INFLUENCE LINES

Fig. 4 shows a few of the influence lines for members of the trusses. It is of interest to note that each member is stressed for all positions of the load, and consequently the work performed in supporting external loads is distributed throughout the entire structure. A second point of interest is that the stress may be positive or negative, depending upon the position of the load. Therefore, for the dead load the stress is the algebraic sum of both, which is less than it would be if the sign were constant. A similar condition holds for a full uniform load. These characteristics of this continuous truss may explain, at least partially, its economy over the simple span types of bridges which are possible for

this site. It was estimated that over 300 tons of steel were saved by the use of this continuous spandrel braced arch instead of a cantilever design with a suspended center span.

DETAILS OF TRUSSES

The trusses, spaced 41 feet apart, are identical and are 782 feet long between end pins and 460 feet long between river piers. They are divided into 34 panels, each 23 feet long, there being 20 panels in the center span and 7 in each of the end spans. The top chord is parallel to the profile of the center line of the road, which is a vertical curve with a 20,000-foot radius. This curvature has four feet for a middle ordinate to a chord drawn between the ends of the abutments. The bottom chords are parabolic in shape and have a rise of one-fifth of the center span, or 92 feet for each span. All chord members are straight between panel points. The web system is a Pratt type. An idea of the size of the truss members may be obtained from some of the cross-sectional areas which follow. The bottom chord member L_7-L_8 , for the first panel out over the river from the pier, has a cross-sectional area of 211 square inches, and is the heaviest member per foot of length in the bridge. The vertical at the pier has an area of 156 square inches. Its length is 108 feet. The top chord at the center has an area of 85 square inches, and the bottom chord has an area of 91 square inches. The remaining members vary from 100 square inches down to 25, the latter being the verticals over the abutments. All truss members were built up of plates and angles, with lacing bars and battens for ties.

Provision was made for secondary stresses where it was found necessary, not only for the members, but for the joints as well. In fabricating the members of the trusses, their lengths were changed by the magnitude of the deformation which resulted from the stresses produced by the dead load plus one-half of the live load, so that the trusses, with these loads applied, will have the same shape and dimensions which were used in their design.

For the wind or lateral bracing, "K" type truss systems were built in the planes of the top and bottom chords, and light cross frames were placed at each panel point. At the abutments, and over the piers, the cross frames were designed sufficiently heavy to carry the reactions from the top chord lateral trusses to the foundations.

FLOOR SYSTEM

The floor system of the bridge was made up of plate girder floor beams framed into the verticals of the trusses, rolled I-beam stringers supported on top of these girders, and a 6½-inch reinforced concrete slab deck carrying a roadway 40 feet wide, and a 6-foot sidewalk. The roadway has a 2-inch bituminous concrete wearing surface. The floor system was so placed relative to the trusses that both trusses are stressed almost the same, making it possible to have the four halves of the two trusses identical, although there is a sidewalk on one side of the bridge which makes the loading unsymmetrical about the center line of construction. Because of the height of the roadway above the river it was thought advisable to have a double curb. This consists of the regular 9-inch reinforced concrete curb, with an additional 11-inch curb placed on top of it but set back 6 inches from the face. In effect, it is equivalent to a 20-inch curb, but eliminates the possibility of damage to the fenders, wheels and other parts of cars. Further protection is furnished by a wrought-iron fence bolted to cast steel supports which were embedded in the concrete. The fence was built up of wrought-iron pipe rails, pales and bars, welded together both for strength and protection against corrosion. Great care was taken to eliminate, as far as possible, all crevices or pockets difficult to paint and where moisture might collect.

BEARINGS

At the ends of the bridge on the abutments, the trusses are supported by forged steel pins 9 inches in diameter which are carried by cast steel shoes bearing on nests of 7-inch steel rollers. These nests are enclosed in steel boxes filled with a heavy gear grease to insure their protection from dirt and water. To facilitate inspection the sides of the grease boxes are made so that they can be taken off and the grease removed. Adjacent to these shoes and rollers are the vertical ties which provide for a possible but highly improbable reversal of reaction, and were used primarily to tie down the ends of the bridge during the cantilever erection. These ties, made up of 22-inch by 1-inch plate with clevises and turnbuckles pinned to them, were connected at their upper ends to the end floor beams, and at their lower ends to plates riveted to steel frames buried in the concrete abutments. A flexible horizontal tie to take the reactions from the lateral "K" trusses was also connected to these abutment plates, and thus relieved the rollers of all but the vertical loads. The maximum positive reaction on the rollers is 513,000

pounds, and to reverse this and bring the ties into action it would be necessary to have all of the live loads simultaneously applied so as to produce only a negative reaction. While this is possible and has been provided for, it is believed that it is quite improbable.

At the piers the trusses were also supported by forged steel pins and cast steel shoes, the latter weighing 15 tons. The pins were 14 inches in diameter, and weighed slightly under 1 ton apiece. Since the piers furnished a horizontal as well as a vertical component for the reaction, the shoes were designed with a horizontal as well as a vertical



FIG. 5.—END TIE AT ABUTMENT

bearing surface. On the Erving or east piers, both surfaces bear on steel beam grillages embedded in the concrete of the piers. Four pairs of anchor bolts hold each shoe rigidly in place. For the Gill or west piers the shoes were carried during erection on a set of rollers which were supported on a steel beam grillage. The horizontal reaction is transmitted to the pier through a pair of 12-inch "H" columns, 3 feet 6 inches on centers, placed horizontally between the back of the shoe and the grillage in the vertical face of the pier. Provision was made for placing a 500-ton jack between these columns. This was used for jacking reactions into the trusses after the erection of the steel was completed.

PIERS AND ABUTMENTS

The piers and abutments are of concrete. The elevation of the shoes at the piers is 2 feet above the highest flood stage of the river. The piers consist of separate footings under each shoe, each having a width parallel to the river of 17 feet, and a depth transverse to the river of 26 feet on the east bank and 34 feet on the west bank, where they extend back to the solid ledge. The depth varies because all of the piers were carried down about 4 feet into bedrock, to withstand any tendency to displacement by flood or ice. The footings on the east bank are above ordinary water level, but on the west side the excavation extended about 6 feet below the water in order to remove loose and soft rock on the surface. The abutment on the east side is 84 feet long and 51 feet wide, and has a height of 60 feet at the bridge seat. It is of cellular construction and has a top slab which serves as a road. The foundations rest upon ledge a few feet below the surface. The rear of the abutment is open and the approach fill extends between as well as around the wings. The westerly abutment is 70 feet long and 20 feet high, and is of the gravity type of construction consisting of gravity walls built up to support the end of the bridge, with pylons extending above the roadway, as on the easterly side. For appearance the Gill pylons were placed 26 feet nearer the river than those on the opposite bank. A simple architectural treatment was used to relieve the monotony of the plain concrete surfaces.

SPECIFICATIONS

All design was made according to the specifications of the American Association of State Highway Officials, dated 1931. The loads provided for were: dead load, uniform and concentrated highway loads for the H-20 loading, impact, sidewalk load, wind or lateral loads, and temperature. The latter was for a variation of 60° above and below 50° Fahrenheit.

CONSTRUCTION OF BRIDGE

There were two contracts for the construction of the bridge, one for the substructures and the other for the superstructure. The substructure contract was awarded to Simpson Brothers Corporation of Boston. No unusual difficulty was experienced in executing this contract. Most of the excavation was to grades above the level of the water in the river. Other than the rock excavation for the Gill piers, which were built upon a shelf cut into the bank, all the excavation that was necessary was the

removal of a few feet of earth covering and a roughening of the surface of the rock. The pouring of the concrete and the placing of the reinforcing steel followed the usual course for such construction. The contractor carried out his various operations on both sides of the river at the same time, with exceptional speed and efficiency, and was able to complete this contract in the five months, beginning in September, 1931, and ending in January, 1932. The weather conditions were extremely favorable during this period, so that only a few days were lost.

ERECTION OF STEEL

In November, 1931, the second contract was awarded to the McClintic-Marshall Corporation, and during the following April work

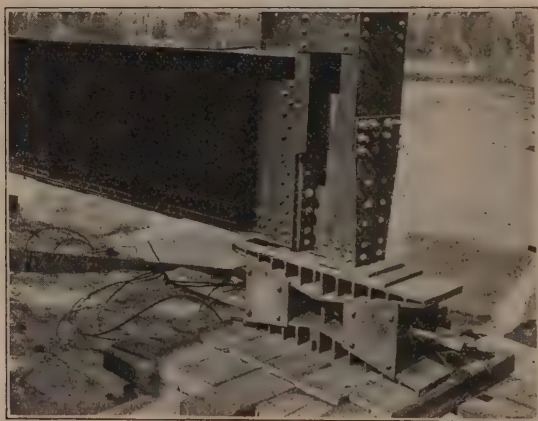


FIG. 6.—ADJUSTABLE STEEL WEDGE JACKS

commenced on the erection of the steel at the Erving end of the bridge. The shore spans were built upon falsework placed at panel points, 2, 4, and 6. This falsework was made up of some of the steel members from the Gill end of the bridge. That for panel point 2 consisted of the floor beam, the two truss verticals, and the sway frame diagonals, all from panel 12. The same pieces from panels 4 and 17 were used at panel points 4 and 6, respectively. These pieces were erected upside down on adjustable steel wedge jacks supported by low timber cribbing, and then the truss steel was built up on special connections on the upper ends of the falsework verticals. After the steel at the piers had been placed, the erection proceeded by cantilevering out over the river, with the ends of the trusses tied down at the abutments. To prevent any

horizontal movement from taking place during erection, the ends of the trusses were also anchored to large "U" bolts embedded in the parapet or vertical walls, directly opposite the end pins.

The traveler was made up of a stiff leg derrick with 35-ton capacity. It had a 75-foot boom with a 5-foot 8-inch gaff attached for a separate line to assist during erection. The weight of the traveler complete was 60 tons. The legs of the derrick were so placed that the steel for erection delivered in back of the traveler was picked up by the boom and

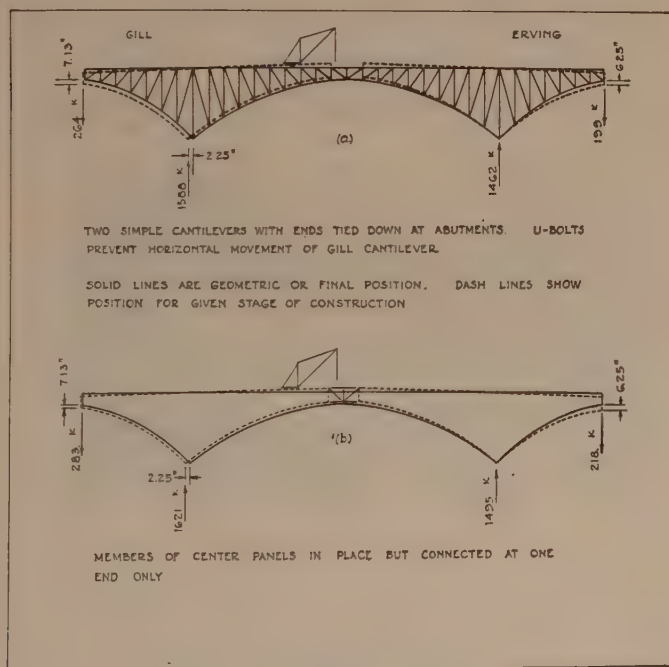


FIG. 7

then swung around and placed in the bridge in front. The traveler was carried on a framework mounted on wheels which ran along a track laid on a plank platform or roadway which was built on the bridge stringers. This roadway was made of 6-inch by 12-inch plank 18 feet long, and was used by trucks hauling the steel from the freight yard to the site. The trucks drove right out on the bridge up to the traveler. Here the traveler unloaded and placed the steel immediately in the bridge, thereby eliminating all storage and rehandling at the site of the work. Wherever possible, the steel was fabricated in two panel lengths,

and erection was planned accordingly; that is, two panels erected, plank roadway laid, traveler advanced two panels, and the next two panels repeated the same cycle. Riveting followed as closely as possible.

When panel 16 had been erected in the Erving half of the bridge, the traveler was rolled back to the abutment again, stopping on the way to remove the falsework at panels 6, 4 and 2. At the abutment it was dismantled and carried around to the Gill side where it was reassembled to continue the erection of the bridge.

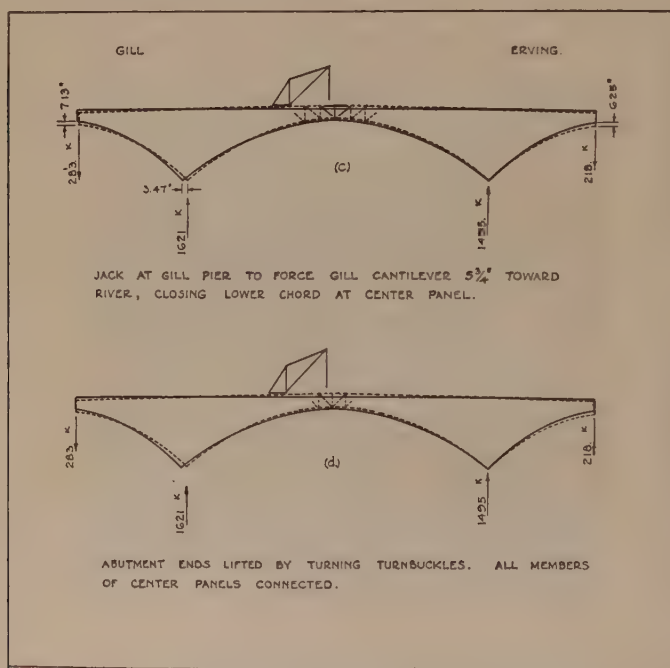


FIG. 8

With but few exceptions, the procedure for the Gill half followed closely that for the Erving. Short timber falsework was used for the shore spans, since the ground was much higher than on the Erving bank. A second difference was the placing of the Gill pier shoes upon roller nests to provide for the horizontal displacements during the jacking operations which were to follow the erection.

When panel 16 was reached and placed, the structure was a pair of simple cantilevers ready to be connected, each cantilever supported at the pier and tied down at the abutment. In Fig. 7, (a) shows the condi-

tions at this stage of the erection, and gives the reactions in Kips, and the deformations or deflections in inches. The solid lines represent the geometric or final shape of the trusses, while the dash lines show the shape as of the stage mentioned. The differences between the reactions and deflections of both the cantilevers are due to the extra load of the erection equipment on the Gill side. At this stage, everything was ready for the placing of the steel for the two center panels. Fig. 7 (b) shows this steel in position, with the chord members supported by the gusset plates of the lateral truss systems and the web members bolted at one end only, the diagonals on the upper end and the vertical on the lower. Complete connection was impossible, since the cantilevers were both erected out of position; that is, the Erving end was built high at the river end and $6\frac{1}{4}$ inches low at the abutment; the Gill side was



FIG. 9. — LIFTING ENDS OF BRIDGE BY TURNBUCKLE

likewise high at the river end and $7\frac{1}{8}$ inches low at the abutment. This was done to provide for the deflections which accompanied the final jacking, and also to insure the connection at the center by lifting the abutment ends rather than drawing them down. The Gill side was also erected $2\frac{1}{4}$ inches back from the normal or final position. Therefore the entire Gill half of the bridge, weighing 1,300 tons, was moved forward $5\frac{3}{4}$ inches until the bottom chords closed at the center. This movement was made by operating the 500-ton jacks at the piers. Conditions at this stage are those in Fig. 8 (c), the bottom chords closed, but the other center panel members still to be connected. To complete the closure of these members it was necessary to lift the ends of the trusses at the abutments by working the turnbuckles so as to lower the center and bring into position the open holes for the rivets. In Fig. 8, (d) shows all of the steel connected at the center panels.

The next step was to place the remainder of the steel in the floor and bracing system. With this completed, the traveler and the plank floor were removed and the Gill turnbuckles were again turned to offset the reduction in load upon the structure. Both halves were then alike in reactions, ties and deflections, as is shown by (e) in Fig. 10. All riveting in the trusses was then completed.

During all of these operations the abutment ends of the trusses were below the final position. To allow for this displacement, the roller

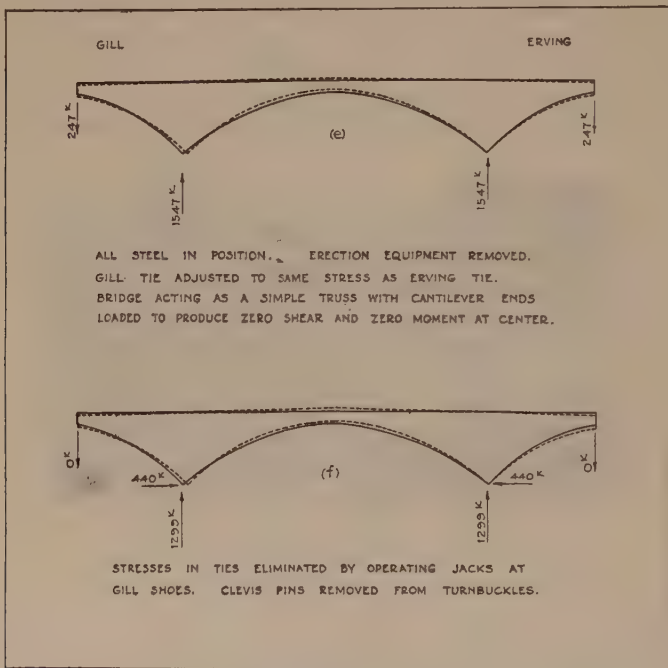


FIG. 10

nests had to be omitted during the erection. However, as they had to be placed before the final jacking occurred, this was the next operation. In order to get them under, the ends of the trusses had to be lifted until they were sufficiently above their normal elevations to give the necessary clearance for sliding the nests and grease boxes into position. This lifting was accomplished by operating the pier jacks until they registered a force of 440 Kips at each of the shoes, which entirely neutralized the stress in the end ties so that the clevis pins could be removed from the eyebolts. Fig. 10 (f) shows this stage. With the ties

inoperative, the pressure in the jacks was reduced until it was only 280 Kips. With this reduction there was an upward deflection of the ends of the bridge to a grade about one-half inch above the normal which was sufficient to permit the rollers in their boxes to be inserted. In Fig. 11, (g) shows the conditions.

The bridge was then ready for the final stressing by jacking the computed reactions into the shoes on both abutments and at the Gill piers. These reactions were those necessary to make the bridge a con-

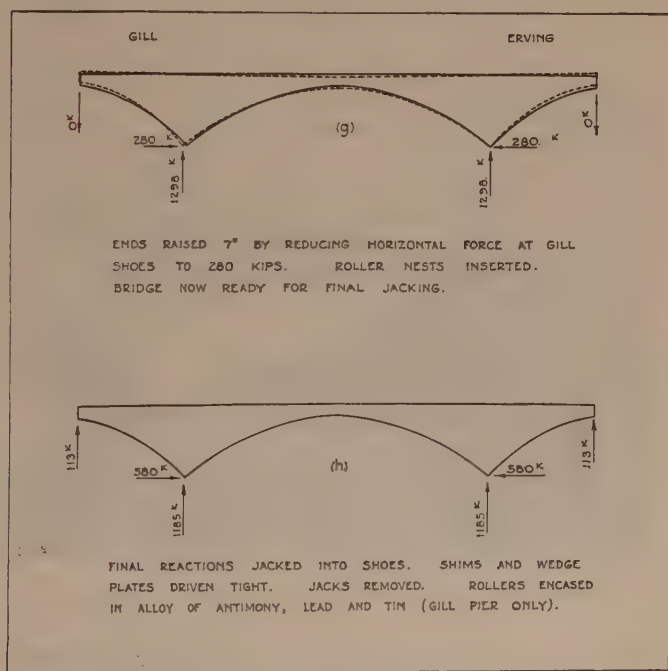


FIG. 11

tinuous spandrel braced arch for the dead load of the steel. There were 100-ton jacks placed under each end of the trusses at the abutments, and these were operated simultaneously with the pier jacks to produce the desired reactions. There were 113 Kips for each of the end jacks, and 580 Kips for each of the pier jacks, all of which is shown in (h) in Fig. 11. All of the reactions computed included a correction for temperature which at the time of these operations was 85 degrees.

To transfer these reactions from the jacks to the shoes, it was necessary to follow immediately with the placing of wedge plates and

shims. At the abutments, tapered wedge plates were driven under the plates carrying the roller nests, until the jacks were relieved of a part of their load, and then pressure was released and the jacks removed. At the piers, shim plates were placed and molten linotype metal was poured to take up all of the play between the ends of the horizontal struts and the grillages buried in the concrete. The jacks were then released and removed. The contractor then proceeded to complete the remaining riveting which was mostly in the stringers and bracing. The ties at the ends of the bridge were again connected, and the turnbuckles tightened and locked by spot welding them to the threads of the bolts.



FIG. 12. — TRAVELER ERECTING STEEL AT CLOSURE

The rollers under the pier shoes were no longer required, and were encased in a block of linotype metal poured in a molten state to a height one inch above their tops. Since the rollers and the linotype metal acted as a unit, the metal was designed to carry some of the reaction. It undoubtedly picked up some of the load of the steel, for the rollers expanded when the linotype metal was poured, and then contracted with the cooling of the entire mass. The linotype metal, with a relatively small coefficient of expansion, remained almost constant in volume, and therefore picked up a part of the load carried by the rollers. Concrete was then poured around the metal blocks and in back of the shoes until all of the struts, plates, etc., were covered up so that at present the shoes have the appearance of being supported by the concrete immediately in back of them.

POURING CONCRETE DECK AND SETTING FENCE

With this part of the work completed, the reinforced concrete slab was poured for the road, curbs and sidewalk. Forms for this concrete were made of ply-wood on wooden joists in sections 4 feet wide and 115 feet long, and were so supported and constructed that the entire form could be dropped between the stringers and pulled ahead and raised against the top flange of the stringers again for the next pouring. This method of formwork saved much time and was economical for the contractor. Five panels or 115 feet of road slab were poured at a time. Pourings alternated at the ends of the bridge. Beginning at the Erving side five panels were poured, and then five were done on the Gill side, and so on until the entire slab was poured.

The fence was then erected and lined up. The cast steel supports buried in the concrete assisted materially in placing and adjusting the fence. Then the bituminous concrete wearing was placed and rolled. During these last operations the steel in the superstructure was painted.

QUANTITIES AND COST

In the construction of this bridge there were used 5,464 cubic yards of rubble concrete; 3,327 cubic yards of reinforced concrete; 251 tons of reinforcing steel; and 2,696 tons of structural steel. The cost of the substructure was \$113,715 and the superstructure \$270,877, making a total of \$384,592.

The McClintic-Marshall Corporation showed exceptional skill in the fabrication and erection of the steelwork. All possible contingencies in the erection were anticipated and provided for, and the workmanship was of a high quality.

The entire work was designed and executed by the Massachusetts Department of Public Works: Frank E. Lyman, Commissioner; Richard K. Hale and Herman A. MacDonald, Associate Commissioners; Arthur W. Dean, Chief Engineer; and George E. Harkness, Bridge Engineer.

OF GENERAL INTEREST

BOSTON SOCIETY OF CIVIL ENGINEERS SCHOLARSHIP IN MEMORY OF DESMOND FITZGERALD AWARDED TO ROLF E. HAMSTROM, STUDENT AT NORTHEASTERN UNIVERSITY .

Rolf E. Hamstrom, a fourth year student in the School of Engineering at Northeastern University, was awarded the Desmond FitzGerald Scholarship of the Boston Society of Civil Engineers on May 17, 1933. The presentation of the scholarship of \$100 was made by Arthur W. Dean, President of the Boston Society of Civil Engineers, at a meeting in Jordan Hall attended by the students of the university.

Mr. Hamstrom, whose home is in Portland, Maine, has maintained an excellent record in his courses and has taken an active part in the affairs of the Northeastern University Section of the Society. The award was recommended by a committee of the University, consisting of Prof. Henry B. Alvord, head of the Department of Civil Engineering, Prof. Winthrop E. Nightingale, and Dean Harold W. Melvin, in accordance

with the conditions of the award established by the Boston Society of Civil Engineers.

Joint Outing, New England Waterworks Association and Boston Society of Civil Engineers

The regular meeting of the Boston Society of Civil Engineers was omitted in June in order to join with the New England Waterworks Association in an outing which was held at the Presidential Country Club at Wayland, Mass., on June 20, 1933.

The usual program of sports, games, bridge and golf was carried out under the direction of the committees. Prizes for winners were distributed at the dinner which was enjoyed by about 200 persons.

PROCEEDINGS OF THE SOCIETY

MINUTES OF MEETINGS

Boston Society of Civil Engineers

MAY 17, 1933 — A regular meeting of the Boston Society of Civil Engineers was held this evening at the Engineers Club, and was called to order by the President, Arthur W. Dean. The attendance was 80. Sixty-six members and guests attended the buffet supper preceding the meeting.

Announcement was made of the election to the Grade of Member of Francis Winon Hamilton.

The President called upon Prof. Albert Haertlein to introduce the speaker, Prof. Lewis Jerome Johnson, professor of civil engineering, Harvard University, who gave a talk on "An Engineer's View of Taxation as affecting the Economic Structure."

Mimeograph copies of the paper were distributed and considerable discussion followed the presentation of his paper. Lantern slide illustrations were used.

Adjourned at 9.30 P.M.

EVERETT N. HUTCHINS, *Secretary*.

Designers Section

MAY 10, 1933. — A meeting of the Designers Section of the Boston Society of Civil Engineers was held on May 10, 1933, at 6.15 P.M. Mr. A. E. Kleinert, Jr., gave a talk on the "Technical Details of the Structural Design of the French King Bridge." He discussed particularly the method of determining stresses in the bridge which was indeterminate to the 3d degree. Typical joint details and the detail design of the abutments were also shown and discussed.

There were 22 members and guests present.

HENRY BRASK, *Clerk*.

ADDITION

Member

HENRY A. MOHR, 1865 Beacon Street,
Waban, Mass.

DEATHS

JOHN WILLIAM LINK	March 28, 1933
ERNEST W. DANFORTH	June 2, 1933
EDWARD F. MILLER	June 12, 1933

BOSTON SOCIETY OF CIVIL ENGINEERS

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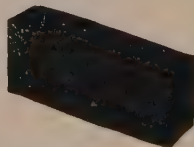
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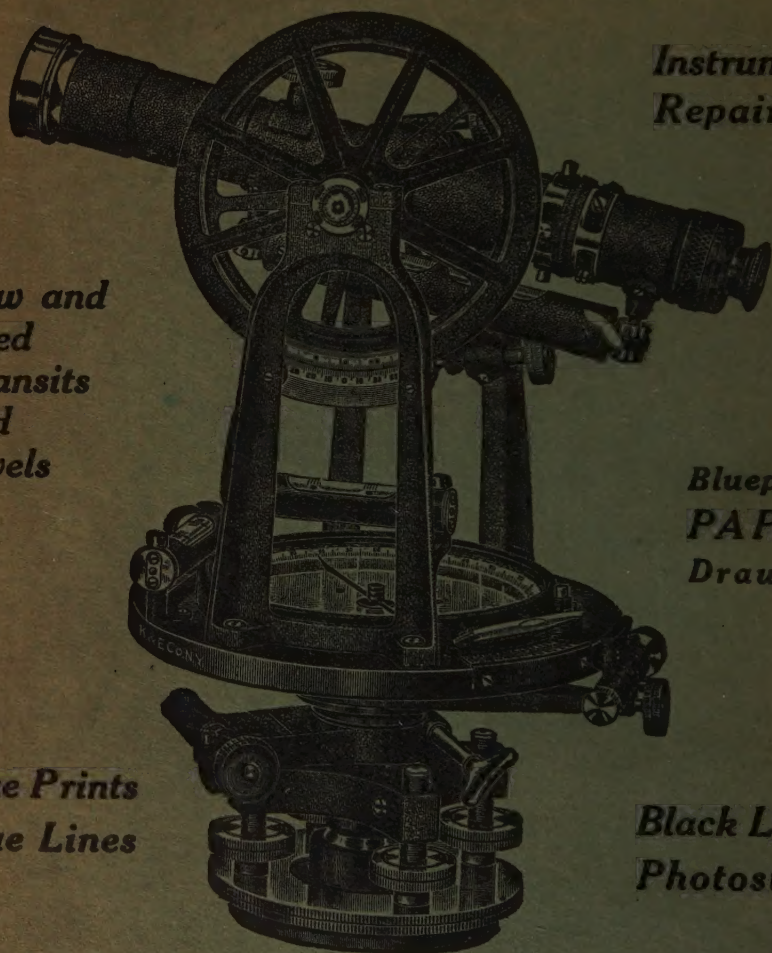
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